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Polyethylene Pipe For Horizontal Directional Drilling

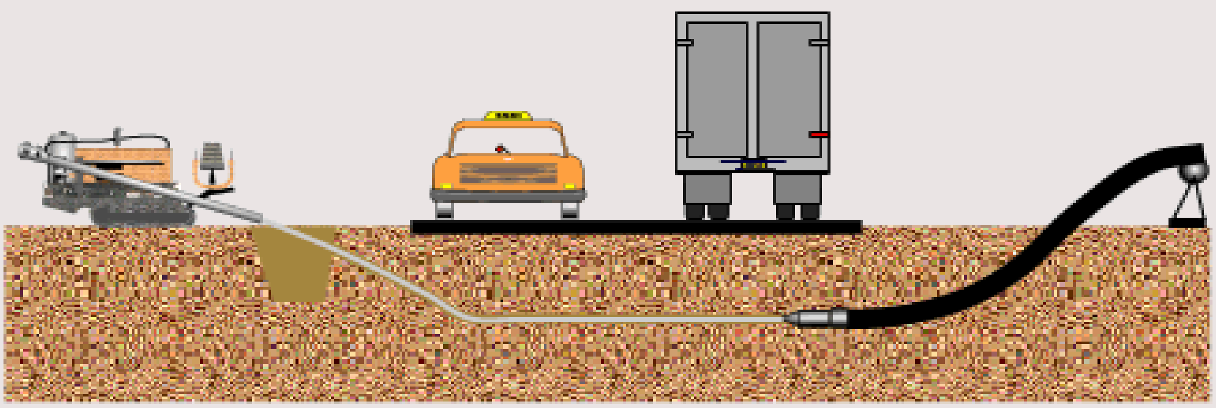

Table of Contents

FIGURES

TABLES

Foreword

Polyethylene Pipe for Horizontal Directional Drilling is one of the chapters being prepared for inclusion in the Plastics Pipe Institute's *PPI Handbook of Polyethylene Piping,* which will be issued as a complete volume in the future. This handbook will cover other uses of polyethylene piping systems including municipal, mining, and industrial applications. Other topics to be addressed in the handbook will include engineering principles, design and installation of polyethylene piping systems, and relevant codes and standards.

PPI is a division of The Society of the Plastics Industry, Inc. (SPI), the major U.S. trade association representing all segments of the plastics industry.

The Municipal and Industrial (M&I) subgroup of PPI are producing the PPI Handbook of Polyethylene Piping. M&I membership consist s of major North American manufacturers of polyethylene (PE) pipe and fittings, PE piping materials, machinery, and equipment used for joining and installing PE piping, related test laboratories, and professional organizations.

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Chapter 11

POLYETHYLENE PIPE FOR HORIZONTAL DIRECTIONAL DRILLING

INTRODUCTION

The Horizontal Directional Drilling (HDD) Industry has experienced so much growth in the past decade, that HDD has become commonplace as a method of installation. One source reported that the number of units in use increased by more than a hundred-fold in the decade following 1984. This growth has been driven by the benefits offered to utility owners (such as the elimination of traffic disruption) and by the ingenuity of contractors in developing this technology. To date, HDD pipe engineering has focused on installation techniques and rightfully so. In many cases, the pipe experiences its maximum lifetime loads during the back-pulling operation.

The purpose of this document is to acquaint the reader with some of the important considerations in selecting the proper polyethylene pipe. Proper selection of pipe involves consideration not only of installation design factors such as pull-back force limits and collapse resistance, but also of the long-term performance of the pipe once installed in the bore-hole. The information herein is not all inclusive; there may be parameters not discussed that will have significant bearing on the proper engineering of an application and the pipe selection. For specific projects, the reader is advised to consult with a qualified engineer to evaluate the project and prepare a specification including design recommendations and pipe selection.

Background

Some of the earliest uses of large diameter polyethylene pipe in directional drilling were for river crossings. These are major engineering projects requiring thoughtful design, installation, and construction while offering the owner the security of deep river bed cover with minimum environmental damage or exposure, and no disruption of river traffic. Polyethylene pipe is suited for these installations because of its scratch tolerance and the fused joining system which gives a zero-leak-rate joint with design tensile capacity equal to that of the pipe.

To date, directional drillers have installed polyethylene pipe for gas, water, and sewer mains; electrical conduits; and a variety of chemical lines. These projects involved not only river crossings but also highway crossings and right-of-ways through developed areas so as not to disturb streets, driveways, and business entrances.

Chapter 11 gives information on the pipe selection and design process. It is not intended to be a primer on directional drilling. The reader seeking such information can refer to the bibliography of this document. A suggested document is the “Mini-Horizontal Directional Drilling Manual” published by the North American Society for Trenchless Technology (NASTT).

HORIZONTAL DIRECTIONAL DRILLING PROCESS

Knowledge of the directional drilling process by the reader is assumed but some review may be of value in establishing common terminology.

Briefly, the HDD process begins with boring a small, horizontal hole (pilot hole) under the crossing obstacle (i.e. a highway) with a continuous string of steel drill rod. When the bore head and rod emerge on the opposite side of the crossing, a special cutter, called a back reamer, is attached and pulled back through the pilot hole. The reamer bores out the pilot hole so that the pipe can be pulled through. The pipe is usually pulled through from the side of the crossing opposite the drill rig.

Pilot Hole

Drilling the pilot hole establishes the path of the drill rod ("drill-path") and subsequently the location of the PE pipe. Typically, the bore-head is tracked electronically so as to guide the hole to a pre-designed configuration. One of the key considerations in the design of the drill-path is creating as large a radius of curvature as possible within the limits of the right-of-way, thus minimizing curvature. Curvature induces bending stresses and increases the pull-back load due to the capstan effect. The capstan effect is the increase in frictional drag when pulling the pipe around a curve due to a component of the pulling force acting normal to the curvature. Higher tensile stresses reduce the pipe's collapse resistance. The drill-path normally has curvature along its vertical profile. Curvature requirements are dependent on site geometry (crossing length, required depth to provide safe cover, staging site location, etc.) But, the degree of curvature is limited by the bending radius of the drill rod and the pipe. For small size pipes the stiff drill rod usually controls the curvature and thus significant bending stresses do not occur in the pipe. The designer should minimize the number of curves and maximize their radii of curvature in the right-of-way by carefully choosing the entry and exit points.

Pilot Hole Reaming

The REAMING operation consists of using an appropriate tool to open the pilot hole to a slightly larger diameter than the carrier pipeline. The percentage oversize depends on many variables including soil types, soil stability, depth, drilling mud, borehole hydrostatic pressure, etc. Normal over-sizing may be from 120% to 150% of the carrier pipe diameter. While the over-sizing is necessary for insertion, it means that the inserted pipe will have to sustain vertical earth pressures without significant side-support from the surrounding soil.

Drilling Mud

Usually a "drilling mud" such as fluid bentonite clay is forced down the hole to stabilize the hole and remove soil cuttings. Drilling mud can be made from clay or polymers. The primary clay for drilling mud is sodium montmorillonite (bentonite). Properly ground and refined bentonite is added to fresh water to produce a "mud". The mud reduces drilling torque, imparts lubrication to the pipe, provides annular flushing of the freshly cut borehole soil debris, and gives stability and support to the bored hole.

Drilling muds are thixotropic and thus thicken when left undisturbed after pull-back. However, unless cementitious agents are added, the thickened mud is no stiffer than very soft clay. Drilling mud provides little to no soil side-support for the pipe.

Pull-back

The pull-back operation involves pulling the entire pipeline length in one segment (usually) back through the drilling mud along the reamed-hole pathway. Proper pipe handling, cradling, bending minimization, surface inspection, and fusion welding procedures need to be followed. Axial tension force readings, constant insertion velocity, mud flow circulation/exit rates, and footage length installed should be recorded. The pullback speed ranges usually between 1 to 2 feet per minute.

Mini-Horizontal Directional Drilling

The Industry distinguishes between mini-HDD and conventional HDD, which is sometimes referred to as maxi-HDD. Mini-HDD rigs can typically handle pipes up to 10" or 12" and are used primarily for utility construction in urban areas, whereas HDD rigs are typically capable of handling pipes as large as 48". These machines have significantly larger pullback forces ranging up to several hundred thousand pounds.

General Guidelines

The designer will achieve the most efficient design for an application by consulting with an experienced contractor and a qualified engineer. Here are some general considerations that may help particularly in regard to site location for PE pipes:

1. Select the crossing route to keep it to the shortest reasonable distance.
2. Find routes and sites where the pipeline can be constructed in one continuous length; or at least in long multiple segments fused together during insertion.
3. Although compound curves have been done, try to use as straight a drill path as possible.
4 Avoid entry and exit elevation differences in excess of 50 feet; both points should be as close as possible to the same elevation.
5. Locate all buried structures and utilities within 10 feet of the drill-path for mini-HDD applications and within 25 feet of the drill-path for maxi-HDD applications. Crossing lines are typically exposed for exact location.
6. Observe and avoid above-ground structures, such as power lines, which might limit the height available for construction equipment.
7. The HDD process takes very little working space versus other methods. However, actual site space varies somewhat depending upon the crossing

distance, pipe diameter, and soil type.

8. Long crossings with large diameter pipe needs bigger, more powerful equipment and drill rig.

9. As pipe diameter increases, large volumes of drilling fluids must be pumped requiring more/larger pumps and mud-cleaning and storage equipment.

10. Space requirements for Maxi-HDD rigs can range from a 100 feet wide by 150 feet long entry plot for a 1000 ft crossing up to 200 feet wide by 300 feet long area for crossing of 3000 or more feet.

11. On the pipe side of the crossing sufficient temporary space should be rented to allow fusing and joining the polyethylene carrier pipe in a continuous string beginning about 75 feet beyond the exit point with a width of 35 to 50 feet depending on the pipe diameter. Space requirements for coiled pipe are considerably less. Larger pipe sizes require larger and heavier construction equipment which need more maneuvering room (use of polyethylene minimizes this though). The initial pipe side “exit” location should be about 50' W x 100" L for most crossings up to 100' W x 150' L for equipment needed in large diameter crossings.

12. Obtain “As-Built” drawings based on the final course followed by the reamer and the installed pipeline. The gravity forces may have caused the reamer to go slightly deeper than the pilot hole and the buoyant pipe may be resting on the crown of the reamed hole. The as-built drawings are essential to know the exact pipeline location and to avoid future third party damage.

GEOTECHNICAL INVESTIGATION

Before any serious thought is given to the pipe design or installation, the designer will normally conduct a comprehensive geotechnical study to identify soil formations at the potential bore sites. The purpose of the investigation is not only to determine if directional drilling is feasible but to establish the most efficient way to accomplish it. With this information the best crossing route can be determined, drilling tools and procedures selected, and the pipe designed. The extent of the geotechnical investigation often depends on the pipe diameter, bore length and the nature of the crossing.

During the survey, the geotechnical consultant will identify a number of relevant items including the following:

a. Soil identification to locate rock, rock inclusions, gravelly soils, loose deposits, discontinuities and hardpan.

b. Soil strength and stability characteristics

c. Groundwater

(Supplemental geotechnical data may be obtained from existing records, i.e. recent nearby bridge constructions, other pipeline/cable crossings in the area.)

For long crossings, borings are typically taken at 700 ft intervals. For short crossings (1000 ft or less), as few as three borings may suffice. The borings should be near the drill-path to give accurate soil data, but sufficiently far from the bore hole to avoid pressurized mud from following natural ground fissures and rupturing to the ground surface through the soil-test bore hole. A rule-of -thumb is to take borings at least 30 ft to either side of bore path. Although these are good general rules, the number, depth and location of boreholes is best determined by the geotechnical engineer.

Geotechnical Data For River Crossings

River crossings require additional information such as a study to identify river bed, depth, stability (lateral as well as scour), and river width. Typically, pipes are installed to a depth of at least 20 ft below the expected future river bottom, considering scour, soil borings for geotechnical investigation are generally conducted to 40 ft below river bottom.

Summary

The best conducted projects are handled by a team approach with the design engineer, bidding contractors and geotechnical engineer participating prior to the preparation of contract documents. The geotechnical investigation is usually the first step in the boring project. Once the geotechnical investigation is completed a determination can be made whether HDD can be used. At that time, design of both the HDPE pipe and the installation can begin.

The preceding paragraphs represent general guidance and considerations for planning and designing an HDD polyethylene pipeline project. These overall topics can be very detailed in nature. Individual HDD contractors and consultant engineering firms should be contacted and utilized in the planning and design stage. Common sense along with a rational in-depth analysis of all pertinent considerations should prevail. Care should be given in evaluating and selecting an HDD contractor based upon successful projects, qualifications, experience and diligence. A team effort, strategic partnership and risk-sharing may be indicated.

PRODUCT DESIGN: DR SELECTION

After completion of the geotechnical investigation and determination that HDD is feasible, the designer turns attention to selecting the proper pipe. The proper pipe must satisfy all hydraulic requirements of the line including flow capacity, working pressure rating, and surge or vacuum capacity. These considerations have to be met regardless of the method of installation. Design of the pipe for hydraulic considerations can be found elsewhere such as in AWWA C906 or the pipe manufacturer's literature and will not be addressed in this chapter. For HDD applications, in addition to the hydraulic requirements, the pipe must be able to withstand (1) pull-back loads which include tensile pull forces, external hydrostatic pressure, and tensile bending stresses, and (2) external service loads (post-installation soil, groundwater, and surcharge loads occurring over the life of the pipeline). Often the load the pipe sees during installation such as the combined pulling force and external pressure will be the largest load experienced by the pipe during its life. The remainder of this document will discuss the DR selection based on pull-back and external service loads. (Polyethylene pipe is classified by DR. The DR is the “dimension ratio” and equals the pipe’s outer diameter divided by the minimum wall thickness.)

While Chapter 11 gives guidelines to assist the designer, the designer assumes all responsibility for determining the appropriateness and applicability of the equations and parameters given in this chapter for any specific application. Directional drilling is an evolving technology and Industry-wide design protocols are still developing. Proper design requires considerable professional judgment beyond the scope of this chapter.

Normally, the designer starts the DR selection process by determining the DR requirement for the internal pressure (or other hydraulic requirements). The designer will then determine if this DR is sufficient to withstand earth, live, and groundwater service loads, if so, then the installation (pull-back) forces are considered. Ultimately, the designer chooses a DR that will satisfy all three requirements; the pressure, the service loads, and the pull-back load.

Although there can be some pipe wall stresses generated by the combination of internal pressurization and wall bending or localized bearing, generally internal pressure and external service load stresses are treated as independent. This is permissible primarily since PE is a ductile material and failure is usually driven by the average stress rather than local maximums, there is a high safety factor applied to the internal pressure, and internal pressurization significantly reduces stresses due to external loads by re-rounding. (One exception to this is internal vacuum, which must be combined with the external pressure.)

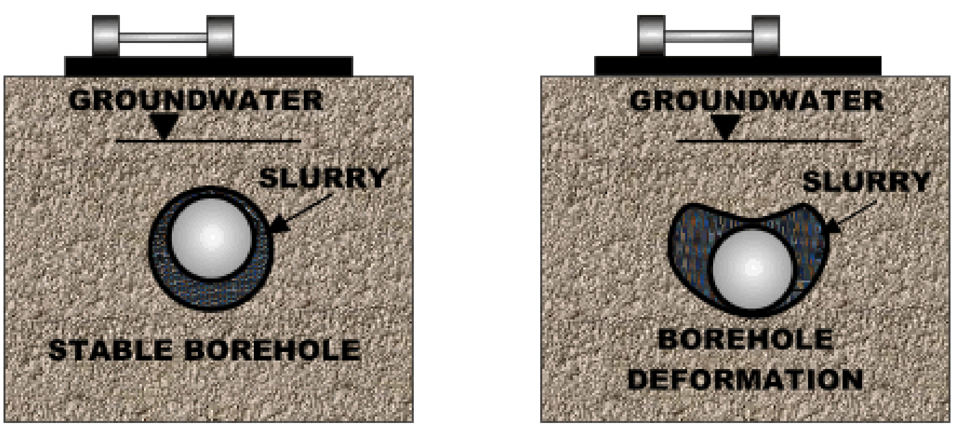

Figure 1 - Borehole Deformation

DESIGN CONSIDERATIONS FOR NET EXTERNAL LOADS

This and the following sections will discuss external buried loads that occur on directional drilled pipes. One important factor in determining what load reaches the pipe is the condition of the borehole, i.e. whether it stays round and open or collapses. This will depend in great part on the type of ground, the boring techniques, and the presence of slurry (drilling mud and cutting mixture). If the borehole does not deform (stays round) after drilling, earth loads are arched around the borehole and little soil pressure is transmitted to the pipe. The pressure acting on the pipe is the hydrostatic pressure due to the slurry or any groundwater present. The slurry itself may act to keep the borehole open. If the borehole collapses or deforms substantially, earth pressure will be applied to the pipe. The resulting pressure could exceed the slurry pressure unless considerable tunnel arching occurs above the borehole. Where no tunnel arching occurs, the applied external pressure is equal to the combined earth, groundwater, and live-load pressure. For river crossings, little arching is anticipated due to the unconsolidated and saturated river bed soils. The applied pressure likely equals the geostatic stress (sometimes called the prism load.) In consolidated soils, arching above the borehole may occur and the applied pressure will likely be less than the geostatic stress, even after total collapse of the borehole crown onto the pipe. If the soil deposit is a stiff clay, cemented, or partially lithified, the borehole may stay open with little or no deformation. In this case, the applied pressure is likely to be just the slurry head or groundwater head.

In addition to the overt external pressures such as slurry head and groundwater, internal vacuum in the pipe results in an increase in external pressure due to the removal of atmospheric pressure from inside the pipe. On the other hand, a positive internal pressure in the pipe may mediate the external pressure. The following equations can be used to establish the net external pressure or, as it is sometimes called, the differential pressure between the inside and outside of the pipe.

Depending on the borehole condition, the net external pressure is defined by either Eq. 1 (deformed/collapsed borehole) or Eq. 2 (open borehole):

$$P_N = P_E + P_{GW} + P_{SUR} - P_I \qquad (1)$$

$$P_N = P_{MUD} - P_I \qquad (2)$$

Where:
- P_N = Net external pressure, psi
- P_E = External pressure due to earth pressure, psi
- P_{GW} = Groundwater pressure (including the height of river water), psi
- P_{SUR} = Surcharge and live loads, psi
- P_I = Internal pressure, psi (negative in the event of vacuum)
- P_{MUD} = Hydrostatic pressure of drilling slurry or groundwater pressure, if slurry can carry shear stress, psi

Earth, ground water, and surcharge pressures used in Eq. 1 are discussed in a following section of this chapter.

$$P_{MUD} = \frac{g_{MUD} H_B}{\frac{inches^2}{ft^2}} \qquad (3)$$

Where: g_{MUD} = Unit weight of slurry (drilling mud and cuttings), pcf
H_B = Elevation difference between lowest point in borehole and entry or exit pit, ft (144 is included for units conversion.)

When calculating the net external pressure, the designer will give careful consideration to enumerating all applied loads and their duration. In fact, most pipelines go through operational cycles that include (1) unpressurized or being drained, (2) operating at working pressure, (3) flooding, (4) shutdowns, and (5) vacuum and peak pressure events. As each of these cases could result in a different net external pressure, the designer will consider all phases of the line's life to establish the design cases.

In addition to determining the load, careful consideration must be given to the duration of each load. PE pipe is viscoelastic, that is, it reacts to load with time-dependent properties. For instance, an HDD conduit resists constant groundwater and soil pressure with its long-term stiffness. On the other hand, an HDD force-main may be subjected to pressure surges resulting in cavitation. When cavitation occurs, the net external pressure equals the sum of the external pressure plus the vacuum. Since cavitation is instantaneous, it is resisted by the pipe's short-term stiffness, which can be four times higher than the long-term stiffness.

For pressure lines, consideration should be given to the time the line sits unpressurized after construction. This may be several months. Most directional drilled lines that contain fluid will have a static head, which will remain in the line once filled. This head may be subtracted from the external pressure due to earth/groundwater load. The designer should keep in mind that the external load also may vary with time, for example, flooding.

EARTH AND GROUNDWATER PRESSURE

Earth loads can reach the pipe when the borehole deforms and contacts the pipe. The amount of soil load transmitted to the pipe will depend on the extent of deformation and the relative stiffness between the pipe and the soil. Earth loading may not be uniform. Due to this complexity, there is not a simple equation for relating earth load to height of cover. Groundwater loading will occur whether the hole deforms or not, the only question is whether or not the slurry head is higher and thus may in fact control design. Thus, what loads reach the pipe will depend on the stability of the borehole.

As the loads reaching the pipe depend on detailed knowledge of the soil, the designer may wish to consult a geotechnical engineer for assistance in determining earth and groundwater loads.

Stable Borehole — Groundwater Pressure Only

A borehole is called stable if it remains round and deforms little after drilling. For instance, drilling in competent rock will typically result in a stable borehole. Stable boreholes may occur in some soils where the slurry exerts sufficient pressure to maintain a round and open hole. Since the deformations around the hole are small, soil pressures transmitted to the pipe are negligible. The external load applied to the pipe consists only of the hydrostatic pressure due to the slurry or the groundwater, if present. Equation 4 gives the hydrostatic pressure due to groundwater or grout. Standing surface water should be added to the groundwater.

$$P_{GW} = \frac{g_W H_W}{144 \dfrac{\text{inches}^2}{\text{ft}^2}} \qquad (4)$$

Where: P_{GW} = Hydrostatic fluid pressure due to ground and surface water, psi

g_w = Unit weight of water, pcf

H_W = Height to free water surface above pipe, ft(144 is included for correct units conversion.)

Borehole Deforms/Collapse With Arching Mobilized

When the crown of the hole deforms sufficiently to place soil above the hole in the plastic state, arching is mobilized. In this state, hole deformation is limited. If no soil touches the pipe, there is no earth load on the pipe. However, when deformation is sufficient to transmit load to the pipe, it becomes the designer's chore to determine how much earth load is applied to the pipe. At the time of this writing, there has been no published reports giving calculation methods for finding earth load on directional drilled pipes. Based on the successful performance of directional drilled PE pipes it is reasonable to assume that some form of arching occurs in many applications. (One noted exception is river crossings.) The designer of HDD pipes may gain some knowledge from the approaches developed for determining earth pressure on auger bored pipes and on jacked pipes. It is suggested that the designer become familiar with all of the assumptions used with these methods.

O'Rourke et. al. published an equation for determining the earth pressure on auger bored pipes assuming a borehole approximately 10% larger than the pipe. In this model, arching occurs above the pipe similar to that in a tunnel where zones of loosened soil fall onto the pipe. The volume of the cavity is eventually filled with soil that is slightly less dense than the insitu soil, but still capable of transmitting soil load. This method of load calculation gives a minimal loading. The method published here is more conservative. It is based on trench type arching as opposed to tunnel arching and is used by Stein to calculate loads on jacked pipe In Stein's model the maximum earth load (effective stress) is found using the modified form of Terzhaghi's equation given by Eq. 6. External groundwater pressure must be added to the effective earth pressure. Stein and O'Rourke's methods should only be considered where the depth of cover is sufficient to develop arching (typically exceeding five (5) pipe diameters), dynamic loads such as traffic loads are insignificant, the soil has sufficient internal friction to transmit arching, and confirmed by a geotechnical engineer. Using the equations given in Stein, the external pressure is given below:

$$P_{EV} = \frac{k g_{SE} H_C}{144 \frac{\text{inches}^2}{\text{ft}^2}} \qquad (5)$$

$$k = \frac{1 - \exp\left(- 2 \frac{K H_C}{B} \tan\left(\frac{d}{2}\right)\right)}{2 \frac{K H_C}{B} \tan\left(\frac{d}{2}\right)} \qquad (6)$$

Where:

P_{EV} = external earth pressure, psi
g_{SE} = effective soil weight, pcf
H_C = depth of cover, ft
k = arching factor
B = "silo" width, ft
d = angle of wall friction, degrees (For HDD, $d = f$)
f = angle of internal friction, degrees
K = earth pressure coefficient given by:

$$K = \tan^2\left(45 - \frac{f}{2}\right)$$

The "silo" width should be estimated based on the application. It varies between the pipe diameter and the borehole diameter. A conservative approach is to assume the silo width equals the borehole diameter. (The effective soil weight is the dry unit weight of the soil for soil above the groundwater level, it is the saturated unit weight less the weight of water for soil below the groundwater level.)

Borehole Collapse with Prism Load

In the event that arching in the soil above the pipe breaks down, considerable earth loading may occur on the pipe. In the event that arching does not occur, the upper limit on the load is the weight of the soil prism ($P_E = \gamma_{SE} H_C$) above the pipe. The prism load is most likely to develop in shallow applications subjected to live loads, boreholes in unconsolidated sediments such as river crossings, and holes subjected to dynamic loads. The "prism" load is given by Eq. 7.

$$P_E = \frac{\gamma_{SE} H_C}{144 \frac{\text{inches}^2}{\text{ft}^2}} \qquad (7)$$

Where:
- P_E = earth pressure on pipe, psi
- γ_{SE} = effective weight of soil, pcf
- H_C = soil height above pipe crown, ft
- (Note: 144 is included for units conversion.)

Combination of Earth and Groundwater Pressure

Where groundwater is present in the soil formation, its pressure must be accounted for in the external load term. For instance, in a river crossing one can assume with reasonable confidence that the directionally drilled pipe is subjected to the earth pressure from the sediments above it combined with the water pressure.

Case(1): Water level at or below ground surface

$$P_E + P_{GW} = \frac{\gamma_B H_W + \gamma_D (H_C - H_W) + \gamma_W H_W}{144 \frac{\text{inches}^2}{\text{ft}^2}} \qquad (8)$$

Case (2): Water level at or above ground surface (i.e. pipe in river bottom)

$$P_E + P_{GW} = \frac{\gamma_B H_C + \gamma_W H_W}{144 \frac{\text{inches}^2}{\text{ft}^2}} \qquad (9)$$

Where: H_W = Height of Ground water above pipe springline, ft
H_C = Height of Cover, ft
g_B = buoyant weight of soil, pcf
g_W = weight of water, pcf
g_D = dry unit weight of soil, pcf

Live Loads

Wheel loads from trucks or other vehicles are significant for pipe buried at shallow depths and they may be significant for shallow HDD pipes. Live load pressures are applied to the pipe only when earth loads are applied. The live load applied to the pipe depends on the vehicle weight, the tire pressure and size, vehicle speed, surface smoothness, pavement and distance from the pipe to the point of loading. In order to develop proper soil structure interaction, pipe subject to vehicular loading should be installed at least 18" or one pipe diameter (whichever is larger) under the road surface. This is assumed to be the case for HDD pipes.

For pipes installed under rigid pavement and subjected to H20 loadings, Table 1 gives the vertical earth pressure at the pipe crown as determined by AISI [3]. Live loads under flexible pavement and unpaved roads can be calculated. (See Spangler and Handy in references.)

Table 1 - H20 Loading Under Rigid Pavement (AISI)

Height of Cover(ft.)	(ft)Load (psf)
1	1800
2	800
3	600
4	400
5	250
6	200
7	175
8	100

The live-load pressure can be obtained from Table 1 by selecting the load based on the height of cover and converting the load to units of “psi” by dividing the load in “psf” by 144.

PERFORMANCE LIMITS

Figure 2 - Performance Limits of HDD Pipe Subjected to Service Loads

PERFORMANCE LIMITS OF HDD INSTALLED PIPE

The design process normally consists of calculating the loads applied to the pipe, selecting a trial pipe DR, then calculating the safety factor for the trial DR. If the safety factor is adequate, the design is complete. If not, the designer selects a lower DR and repeats the process. The safety factor is established for each performance limit of the pipe by taking the ratio of the pipe's ultimate strength or resistance and the applied load.

Typically, polyethylene pipe is installed in a bore hole 1.25 to 1.5 times larger in diameter than the pipe itself. Drilling mud and cuttings fill the annular space. This mixture is thixotropic, but with the exception of cementatious grouts its viscosity is at best similar to very soft clay. It does not provide soil support for the pipe as does pipe embedment material. Therefore, the designer normally ignores any support from the annular space mixture and selects pipe, which has sufficient ring stiffness to resist the net external pressure without support of the surrounding soil. External pressure applied to the HDD pipe produces (1) a compressive ring thrust in the pipe wall and (2) ring bending deflection. Understanding the consequences of the compressive thrust and the bending are essential to designing pipe for HDD applications. The performance limit of unsupported PE pipe subjected to compressive thrust is ring buckling (collapse). The performance limit of a PE pipe subjected to ring bending (a result of non-uniform external load, i.e. earth load) is ring deflection.

Time-Dependent Behavior

The performance limits for PE pipe in directional drilled applications are dependent on either the modulus of elasticity or the tensile strength. Both of these properties are time-dependent. PE pipe's resistance to a newly applied load increment decreases with time as the molecular structure rearranges due to viscoelasticity. This results in a higher resistance to short-term loading than to long-term loading. Careful consideration must be given to the duration and frequency of each load, so that the performance limit associated with that load can be calculated using PE material properties representative of that time period. For instance, during pull-back, the pipe's tensile yield strength decreases with pulling time, so the safe (allowable) pulling stress is a function of time. See Table 2 for typical values for high density PE (HDPE) and medium density PE (MDPE).

For viscoelastic materials, the ratio of the applied stress to strain is called the apparent modulus of elasticity, because the ratio varies with load rate. Typical values for the apparent modulus of elasticity at 73°F (23°C) are presented in Table 2. Consult the manufacturer for specific applications.

Table 2 - Apparent Modulus of Elasticity and Safe Pull Tensile Stress @ 73° F

Typical Apparent Modulus of Elasticity			Typical Safe Pull Stress		
Duration	HDPE	MDPE	Duration	HDPE	MDPE
Short-term	110,000 psi (800 MPa)	87,000 psi (600 MPa)	30 min	1300 psi (9.0 MPa)	1000 psi (6.9 MPa)
10 hours	57,500 psi (400 MPa)	43,500 psi (300 MPa)	60 min	1200 psi (8.3 MPa)	900 psi (6.2 MPa)
100 hours	51,200 psi (350 MPa)	36,200 psi (250 Mpa)	12 hours	1150 psi (7.9 Mpa)	850 psi (5.9 Mpa)
50 years	28,200 psi (200 Mpa)	21,700 psi (150 Mpa)	24 hours	1100 psi (7.6 MPa)	800 psi (5.5 MPa)

RING DEFLECTION (OVALIZATION)

Non-uniform pressure acting on the pipe's circumference causes bending deflection of the pipe ring. Normally, the deflected shape is that of an oval. Ovalization may occur in non-rerounded coiled pipe and to a lesser degree in straight lengths that have been stacked but the primary sources of ovalization of directional drilled pipes are (1) buoyant deflection due to the fluid in the borehole and (2) ring deflection due to earth load in a deformed or collapsed borehole. Ovalization may also occur during pullback due to bending the pipe around a curved path in the borehole. Ovalization reduces the pipes hydrostatic collapse resistance and creates tensile bending stresses in the pipe wall. It is normal and expected for buried PE pipes to undergo ovalization (or as it is often called "ring deflection"). Proper design and installation will limit ovalization to prescribed values so that it has no adverse effect on the pipe.

Ring Deflection Due To Buoyancy

A pressure difference occurs when pipe is submerged in grout due to the difference in grout head between the invert and the crown of the pipe. The pressure difference applies a force, which deflects the invert upward toward the crown, thus creating ovality. This deflection is referred to as “buoyant deflection”. The resistance to buoyant deflection is given by Equation 10.

$$\frac{D}{D} = \frac{0.1169\ W\left(\frac{D}{2}\right)^4}{EI} \qquad (10)$$

Where: D = ring deflection, in
D = pipe diameter, in
g_W = weight of fluid in borehole, lbs/in^3
E = modulus of elasticity, psi
I = moment of inertia of pipe wall cross-section ($t^3/12$), in^4/in
(To convert fluid weight from lbs/ft^3 to lbs/in^3 divide by 1728.)

Ring Deflection Due To Earth Load

The earth load applied to directional drilled pipes depends on insitu soil characteristics and borehole stability. Methods for calculating estimated earth loads, when they occur, are given in the previous section on “Earth and Groundwater Pressure”. Generally, earth load is applied to the pipe crown with a reaction occurring at the invert. As slurry provides essentially no side-support, there is little pressure at the springline to restrain vertical deflection. The primary resistance to deflection is provided by the pipe’s stiffness.

At the time of this writing, there is no mathematical expression relating an HDD pipe’s ring deflection to the earth load. Formulas used for entrenched pipe are likely not suited. Spangler’s Iowa formula is difficult to apply as the slurry stiffness (E’) is not known and often may be nearly zero. Since the pipe will be the major load carrying element, the customary practice of using the short-term pipe stiffness in the Iowa formula is unconservative. Pipe creep will govern deflection and therefore the long-term stiffness should be used.

The pipe stiffness equation (also called the parallel-plate load equation) is one reasonable candidate for determining ring deflection, but it is based on a point load at the crown and invert. Whereas actual soil loads will be applied over a good portion of the top and bottom halves of the pipe. Watkins and Anderson give two ring deflection formulas for uniform loading on the top half of a pipe in the Appendix of their text. One formula assumes the pipe’s invert is supported on a rigid, flat base while the other assumes the invert reaction load is uniform around the bottom half of the pipe. Neither case accurately models what occurs, as there is likely to be some settlement of the bottom of the pipe into the insitu soil. Equation 11 is based on using a coefficient that is the average of the two formulas. (Coincidentally, Equation 11 gives the same deflection as the Iowa formula with an E’ of zero.)

$$\frac{D}{D} = \frac{0.0125P_E}{\dfrac{E}{12\,(DR - 1)^3}} \qquad (11)$$

Where:
D = ring deflection, in
D = pipe diameter, in
P_E = Earth pressure, psi
DR = Pipe Dimension Ratio
E = modulus of elasticity, psi

Ring Deflection Limits (Ovality Limits)

The limiting ovalization or ring deflection (in percent) is determined by the strain occurring in the pipe wall, the pipe's hydraulic capacity, and the geometric stability of the pipe. Jansen observed that for PE, pressure-rated pipe, subjected to soil pressure only, "no upper limit from a practical design point of view seems to exist for the bending strain" [7]. On the other hand, pressurized pipes are subject to strains from both soil induced deflection and internal pressure. The combined strain may produce a high, localized outer-fiber tensile stress. However, as the internal pressure is increased the pipe tends to re-round and the bending strain is reduced. Due to this potential for combined strain (bending and hoop tensile), it is conservative to limit deflection of pressure pipes to less than non-pressure pipes. In lieu of an exact calculation for allowable deflection limits, the limits in Table 3 can be used.

Table 3
Design Deflection Limits of Buried Polyehtylene Pipe, Long Term, %*

DR or SDR	21	17	15.5	13.5	11	9	7.3
Deflection Limit (%Dia) Non-Pressure Applications	7.5	7.5	7.5	7.5	7.5	7.5	7.5
Deflection Limit (%Dia) Pressure Appliations	7.5	6.0	6.0	6.0	5.0	4.0	3.0

*Deflection limits for pressure applications are equal to 1.5 times the short-term deflection limits given in TableX2.1 of ASTM F-714.

Design deflections are for use in selecting DR and for field quality control. (Field measured deflections exceeding the design deflection do not necessarily indicate unstable or over-strained pipe. In this case, an engineering analysis of such pipe should be performed before acceptance.)

Unconstrained Buckling

The application of a uniform external pressure to the pipe creates a ring compressive hoop stress in the pipe's wall. Buckling occurs as the compressive hoop stress in the wall increases and the pipe reaches a point of instability where increasing the stress causes a sudden and large inward deformation of the wall. Constraining the pipe by embedding it in soil or cementatious grout will increase the pipe's buckling strength. However, drilling mud or slurry are not sufficiently stiff to provide such support. The following equation, known as Levy's equation, may be used to determine the allowable external pressure (or negative internal pressure) for a HDD pipe in a non-cementatious slurry or grout:

$$P_{UA} = \frac{2E}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 \frac{f_o}{N} \qquad (12)$$

Where: P_{ua} = Allowable unconstrained pressure, psi
E = Modulus of elasticity (apparent), psi
μ = Poisson's Ratio
Long term loading - 0.45
Short term loading - 0.35
DR = Dimension ratio (D_o/t)
f_o = ovality compensation factor (see figure 3)
N = Safety factor, generally 2.0 or higher

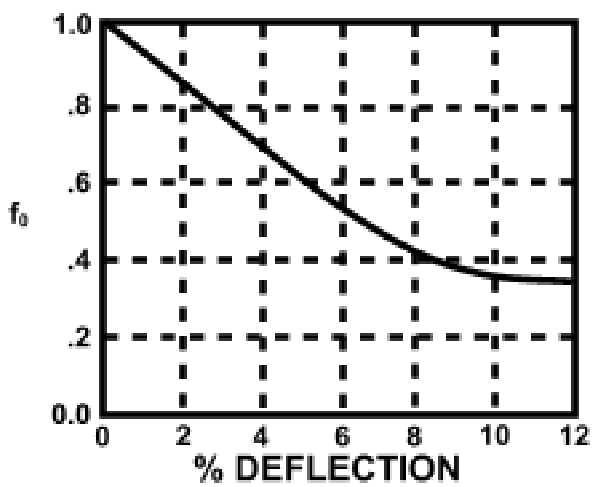

Figure 3 - Ovality Compensation Factor

Note that the modulus of elasticity and Poisson's ratio are a function of the duration of the anticipated load. If the safety factor in Levy's equation is set equal to one, the equation gives the critical buckling pressure for the pipe. Table 4 below gives values of the critical buckling (collapse) pressure for different DR's of PE pipe. For design purposes, the designer must reduce the values by a safety factor and by ovality compensation. When using this table for determining pipe's resistance to buckling during pullback and additional reduction for tensile stresses is required, which is discussed in a later section of this chapter.

Table 4 - Critical Buckling (Collapse) Pressure for unconstrained HDPE Pipe* @73^{0}F

Service Life	Pipe Units	SDR 7.3	9	11	13.5	15.5	17	21
Short-term	psi	1003	490	251	128	82	61	31
	ft H_2O	2316	1131	579	297	190	141	72
	in Hg	2045	999	512	262	168	125	64
100 hrs	psi	488	238	122	62	40	30	15
	ft H_2O	1126	550	282	144	92	69	35
	in Hg	995	486	249	127	82	61	31
50 yrs	psi	283	138	71	36	23	17	9
	ft H_2O	653	319	163	84	54	40	20
	in Hg	577	282	144	74	47	35	18

(Table does not include ovality compensation or safety factor.)

* Full Vacuum is 14.7 psi, 34 ft water, 30 in Hg.
* Multipliers for Temperature Rerating:

*Axial Tension during pull-back reduces collapse strength.

60^{0}F (16^{0}C)	73.4^{0}F (23^{0}C)	100^{0}F (38^{0}C)	120^{0}F (49^{0}C)
1.08	1.00	0.78	0.63

Wall Compressive Stress

The compressive stress in the wall of a directional drilled PE pipe rarely controls design and it is normally not checked. However, it is included here because in some special cases such as directional drilling at very deep depths such as in landfills it may control design.

The earth pressure applied to a buried pipe creates a compressive thrust stress in the pipe wall. When the pipe is pressurized the stress is reduced due to the internal pressure creating tensile thrust stresses. The net stress can be positive or negative depending on the depth of cover. Buried pressure lines may be subject to net compressive stress when shut down or when experiencing vacuum. These are usually short term conditions and are not typically considered significant for design, since the short term design stress of polyolefins is considerably higher than the long term design stress. Pipes with large depths of cover and operating at low pressures may have net compressive stresses in the pipe wall. The following equation can be used to determine the net compressive stress:

$$S_C = \frac{P_S D_O}{288t} - \frac{PD}{2t} \qquad (13)$$

Where:
S_C = Compressive wall stress, psi
P_S = Earth load pressures, psf
D_O = Pipe outside diameter, in
t = Wall thickness, in
P = (Positive) internal pressure, psi
D = Mean diameter, D_O-t, in

The compressive wall stress should be kept less than the allowable compressive stress of the material. A conservative approach is to assume that the allowable compressive stress is equal to the allowable hydrostatic design stress. For PE3408 HDPE pipe grade resins, 800 psi is a safe allowable stress.

EXAMPLE CALCULATIONS

An example calculation for selecting the DR for a HDD pipe is given in the Appendix.

INSTALLATION DESIGN CONSIDERATIONS

After determining the DR required for long-term service the designer must determine if this DR is sufficient for installation. Since installation forces are so significant, a lower DR (stronger pipe) may be required. Proper installation procedures may reduce some of these forces to an inconsequential level.

During pull-back the pipe is subjected to axial tensile forces caused by the frictional drag between the pipe and the borehole or slurry, the frictional drag on the ground surface, the capstan effect around drill-path bends, and hydrokinetic drag. In addition, the pipe may be subjected to external hoop pressures due to net external fluid head and bending stresses. The pipe's collapse resistance to external pressure given in Levy's equation is reduced by the axial pulling force. Furthermore, the drill path curvature may be limited by the pipe's bending radius. (Torsional forces occur but are usually negligible when back-reamer swivels are properly designed.) Considerable judgment is required to predict the pull-back force because of the complex interaction between pipe and soil. Sources for information include experienced drillers and engineers, programs such as DRILLPATH (1) and publications. Typically, pull-back force calculations are approximations that depend on considerable experience and judgment.

Because of the large number of variables involved and the sensitivity of pull-back forces to installation techniques, the formulas presented in this document are for guidelines only and are given only to familiarize the designer with the interaction that occurs during pullback. Pull-back values obtained should be considered only as qualitative values and used only for preliminary estimates. The designer is advised to consult with an experienced driller or with an engineer familiar with calculating these forces.

PULL-BACK FORCE

Large HDD rigs can exert between 100,000 lbs. to 200,000 lbs. pull force. The majority of this power is applied to the cutting face of the reamer device/tool, which precedes the pipeline segment into the borehole. It is difficult to predict what portion of the total pull-back force is actually transmitted to the pipeline being inserted.

The pulling force which overcomes the combined frictional drag, capstan effect, and hydrokinetic drag, is applied to the pull-head and first joint of HDPE pipe. The axial tensile stress grows in intensity over the length of the pull. The duration of the pull-load is longest at the pull-nose. The tail end of the pipe segment has zero applied tensile stress for zero time. The incremental time duration of stress intensity along the length of the pipeline from nose to tail causes a varying degree of recoverable elastic strain and viscoelastic stretch per foot of length along the pipe.

The DR must be selected so that the tensile stress due to the pull-back force does not exceed the permitted tensile stress for the pipe. Increasing the pipe wall thickness will allow for a greater total pull-force, but the thicker wall also increases the weight per foot of the pipe in direct proportion. Hence, thicker wall pipe may not necessarily reduce stress, only increase the absolute value of the pull force or tonnage. The designer should carefully check all proposed DR's.

Frictional Drag Resistance

Pipe resistance to pull-back in the borehole depends primarily on the frictional force created between the pipe and the borehole or the pipe and the ground surface in the entry area, the frictional drag between pipe and drilling slurry, the capstan effect at bends, and the weight of the pipe. Equation 14 gives the frictional resistance or required pulling force for pipe pulled in straight, level bores or across level ground.

$$F_P = \mu w_B L \qquad (14)$$

Where:
F_P = pulling force, lbs
μ = coefficient of friction between pipe and slurry (typically 0.25) or between pipe and ground (typically 0.40)
w_B = net downward (or upward) force on pipe, lb/ft
L = length, ft

When a slurry is present, w_B, is the upward buoyant force of the pipe and its contents. Filling the pipe with fluid significantly reduces the buoyancy force and thus the pulling force. Polyethylene pipe has a density near that of water. If the pipe is installed "dry" (empty) using a closed nose-pull head, the pipe will want to "float" on the crown of the borehole leading to the sidewall loading and frictional drag through the buoyancy-per-foot force and the wetted soil to pipe coefficient of friction. If the pipe is installed full of water, or better yet, full of drilling mud using an open-end pulling-head, the net buoyant force is drastically reduced (see the calculation examples). The overall frictional drag force is reduced to less than about 10% of the weight of the polyethylene pipe or just a few percent of the buoyant (empty) pipe frictional drag.

[Note that the buoyant force pushing the empty pipe to the borehole crown will cause the PE pipe to "rub" the borehole crown. During pull-back, the moving drill mud lubricates the contact zone. If the drilling stops, the pipe stops, or the mud flow stops, the pipe, slightly ring deflected by the buoyant force, can push up and squeeze out the lubricating mud. The resultant "start-up" friction is measurably increased. The pulling load to loosen the PE pipe from being "stuck" in the now decanted (moist) mud can be very high. This situation is best avoided by using higher ring stiffness pipes, inserting full pipe not empty pipe, and continuous drilling progress.]

Capstan Force

For curves in the borehole, the force can be factored into horizontal and vertical components. Huey et al.(3) shows an additional frictional force that occurs in steel pipe due to the pressure required by the borehole to keep the steel pipe curved. For bores with a radius of curvature similar to that used for steel pipe, these forces are likely insignificant for PE pipe. For very tight bends, it may be prudent to consider them. The frictional resistance during a pull is compounded by the capstan effect. As the pipe is pulled around a curve or bend creating an angle q, there is a compounding of the forces due to the direction of the pulling vectors. The pulling force, F_C, due to the capstan effect is given in Eq. 15. Equations 14 and 15 are applied recursively to the pipe for each section along the pull-back distance as shown in Figure 4. This method is credited to Larry Slavin of Bellcore (Middletown, NJ).

$$FC = e^{mq}(m\, w_B L) \qquad (15)$$

Where: e = Natural logarithm base (e=2.71828)
m = coefficient of friction
q = angle of bend in pipe, radians
w_B = weight of pipe or buoyant force on pipe, lbs/ft
L = Length of pull, ft

$$F_1 = \exp(m_g \cdot a) \cdot [m_g \cdot w_P (L_1 + L_2 + L_3 + L_4)]$$

$$F_2 = \exp(\mu_b \cdot \alpha) \cdot (F_1 + \mu_b \cdot w_b \cdot L_2 + w_b \cdot H - \mu_g \cdot w_p \cdot L_2 \cdot \exp(\mu_g \cdot \alpha))$$

$$F_3 = F_2 + \mu_b \cdot w_b \cdot L_3 - \exp(\mu_b \cdot \alpha) \cdot (\mu_g \cdot w_p \cdot L_3 \cdot \exp(\mu_g \cdot \alpha))$$

$$F_4 = \exp(\mu_b \cdot \beta)\,[F_3 + \mu_b \cdot w_b \cdot L_4 - w_b \cdot H - \exp(\mu_b \cdot \alpha) \cdot (\mu_g \cdot w_p \cdot L_4 \cdot \exp(\mu_g \cdot \alpha))]$$

Where:
- H = Depth of bore
- F_i = Pull Force on pipe at Point i
- L_i = Horizontal distance of Pull from point to point
- μ = Coeff. of friction (ground (g) and borehole (b))
- w = Pipe weight (p) and Buoyant pipe weight (b)
- α, β = Entry and Exit angles

Figure 4
Estimated Pull-back Force Calculation

Hydrokinetic Force

During pulling, pipe movement is resisted by the drag force of the drilling fluid. This hydrokinetic force is difficult to estimate and depends on the drilling slurry, slurry flow rate, and borehole and pipe sizes. Typically, the hydrokinetic pressure is in the 30 to 60 kPa (4 to 8 psi) range.

$$F_{HK} = p\,\frac{\pi}{8}\left(D_H^{\,2} - OD^2\right) \tag{16}$$

Where:
- F_{HK} = hydrokinetic force, lbs
- p = hydrokinetic pressure, psi
- D_H = borehole diameter, in
- OD = pipe outside diameter, in

The total pull back force, F_T, then is the combined pull-back force, F_P, plus the hydrokinetic force, F_{HK}. For the example shown in Figure 4, F_P equals F_4.

TENSILE STRESS DURING PULL-BACK

The maximum outer fiber tensile stress should not exceed the safe pull stress. The maximum outer fiber tensile stress is obtained by taking the sum of the tensile stress in the pipe due to the pull-back force, the hydrokinetic pulling force, and the tensile bending stress due to pipe curvature. During pull-back it is advisable to monitor the pulling force and to use a “weak link” (such as a pipe of higher DR) or other failsafe method to prevent over-stressing the pipe.

The tensile stress occurring in the pipe wall during pull-back is given by Eq. 17.

$$s_T = \frac{F_T}{p\ t\ (D_{OD} - t)} + \frac{E_T D_{OD}}{2R} \qquad (17)$$

Where: s_T = Axial tensile stress, psi
F_T = Total pulling force, lbs
t = Minimum wall thickness, in
D_{OD} = Outer diameter of pipe, in
E_T = Time-dependent tensile modulus, psi
R = Minimum radius of curvature in bore path, in

The axial tensile stress due to the pulling forces should not exceed the safe pull load. Values in Table 5 can be used or the designer can calculate a safe pull load based on a different pull time. As discussed in a previous section, the tensile strength of PE pipe is load-rate sensitive and therefore values of "safe" pull loads which might be satisfactory for slip lining or insert renewal where the pull load is imposed for a maximum of 30 min. to 60 min may not be satisfactory for directional drilling. With directional drilling, the time duration of stress intensity may be longer between 4 hours to 24 hours. The "safe" pull-load is time dependent. Hence, the 60 min. or less "safe" pull load (to limit elongation in the forward portion of the pipeline where the pull force is largest), is inappropriate for longer duration pulls. Table 5 gives safe tensile stress values for time intervals. The 24 hour value will normally keep the pull-nose "stretch" low and avoid localized herniation of the HDPE pipeline. Allowable safe pull-back values for gas pipe are given in ASTM F-1807, "Practice for Determining Allowable Tensile Load for Polyethylene (PE) Gas Pipe during Pull-In Installation".

After pull back, pipe may take several hours (typically equal to the duration of the pull) to recover from the axial strain. When pulled from the reamed borehole, the pull-nose should be pulled out about 3% longer than the total length of the pull. The elastic strain will recover immediately and the viscoelastic stretch will "remember" its original length and recover overnight. One does not want to come back in the morning to discover the pull-nose sucked back below the borehole exit level due to stretch recovery and thermal-contraction to an equilibrium temperature. In the worst case, the driller may want to pull out about 4% extra length (40 feet per 1000 feet) to insure the pull-nose remains extended beyond the borehole exit.

Table 5. Safe Pull Load for HDPE Pipes

Pipe Safe	Pull Load @ 24 hrs
1-1/4" SDR 11	800 lbs
2" SDR 11	1,600 lbs
4" SDR 11	6,000 lbs
8" SDR 17	14,000 lbs
8" SDR 11	21,000 lbs
12" SDR 17	31,000 lbs
12" SDR 11	46,000 lbs
24" SDR 17	110,000 lbs
24" SDR 11	164,000 lbs
36" SDR 17	248,000 lbs

EXTERNAL PRESSURE DURING INSTALLATION

During pull-back it is reasonable to assume that the borehole remains stable and open and that the borehole is full of drilling slurry. The net external pressure due to fluid in the borehole, then, is the slurry head, P_{MUD}. This head can be offset by pulling the pipe with an open nose or filling the pipe with water for the pull-back. However, this may not always be possible, for instance when installing electrical conduit. In addition to the fluid head in the borehole, there are also dynamic sources of external pressure:

1. If the pulling end of the pipe is capped, a plunger action occurs during pulling which creates a mild surge pressure. The pressure is difficult to calculate. The pipe will resist such an instantaneous pressure with its relatively high short-term modulus. If care is taken to pull the pipe smoothly at a constant speed, this calculation can be ignored. If the pipe nose is left open, this surge is eliminated.

2. External pressure will also be produced by the frictional resistance of the drilling mud flow. Some pressure is needed to pump drilling mud from the reamer tool into the borehole, then into the pipe annulus, and along the pipe length while conveying reamed soil debris to the mud recovery pit. An estimate of this short term hydrokinetic pressure may be calculated using annular flow pressure loss formulas borrowed from the oil well drilling industry. This external pressure is dependent upon specific drilling mud properties, flow rates, annular opening, and hole configuration. This is a short term installation condition. Thus, HDPE pipe's short term external differential pressure capabilities are compared to the actual short term total external pressure during this installation condition.

Under normal conditions, the annular-flow back pressure component is less than 4-8 psi.

In consideration of the dynamic or hydrokinetic pressure, P_{HK}, the designer will add additional external pressure to the slurry head:

$$P_N = P_{MUD} + P_{HK} - P_I \qquad (18)$$

Where the terms have been defined previously.

Resistance to External Collapse Pressure During Pull-Back Installation

The allowable external buckling pressure equation, Eq.12, with the appropriate time-dependent modulus value can be used to calculate the pipe's resistance to the external pressure, P_N, given by Eq. 18 during pull-back. The following reductions in strength should be taken:

1. If the pipe is empty, the buoyant force will cause a reduction in the vertical diameter. The deflection fraction due to buoyancy can be calculated using Eq. 10. This value can be converted to percent ovality by multiplying the deflection fraction by 100. Ovality reduces buckling resistance. The appropriate ovality compensation factor for Eq.12 is found in Fig 3.

2. The tensile pulling force reduces the buckling resistance. This can be accounted for by an additional reduction factor, f_R. The pulling load in the pipe creates a hoop strain as described by Poisson's ratio. The hoop strain reduces the buckling resistance. Multiply Eq. 12 by the reduction factor, f_R to obtain the allowable external buckling pressure during pull-back.

$$f_R = \sqrt{(5.57 - (r + 1.09)^2)} - 1.09 \qquad (19)$$

$$r = \frac{s_T}{2s} \qquad (20)$$

Where s_T = calculated tensile stress during pull-back (psi)
s = safe pull stress (psi)

Since the pull-back time is typically several hours, a modulus value consistent with the pull-back time can be selected from Table 2.

BENDING STRESS

HDD river crossings incorporate radii-of-curvature, which allow the HDPE pipe to cold bend within its elastic limit. These bends are so long in radius as to be well within the flexural bending capability of polyethylene pipe. PE3408 of SDR 11 can be cold bent to 25 times its nominal OD (example: for a 12" SDR 11 HDPE pipe, the radius of curvature could be from infinity down to the minimum of 25 feet, i.e., a 50-foot diameter circle). Because the drill stem and reaming rod are less flexible, normally polyethylene can bend easily to whatever radius the borehole steel drilling and reaming shafts can bend because these radii are many times the pipe OD. However, in order to minimize the effect of ovaling some manufacturers limit the radius of curvature to a minimum of 40 to 50 times the pipe diameter. As in a previous section, the tensile stress due to bending is included in the calculations.

THERMAL STRESSES AND STRAINS

HDD pipeline river crossings are considered to be fully restrained in the axial direction by the friction of the surrounding soil. This is generally accepted to be the case, though, based on uncased borings through may soil types with the progressive sedimentation and borehole reformation over a few hours to several months. This assumption is valid for the vast majority of soil conditions, although it may not be completely true for each and every project. During pipe installation, the moving pipeline is not axially restrained by the oversize borehole. However, the native soil tends to sediment and embed the pipeline when installation velocity and mud flow are stopped, thus allowing the soil to grip the pipeline and prevent forward progress or removal. Under such unfortunate stoppage conditions, many pipelines may become stuck within minutes to only a few hours.

The degree to which the pipeline will be restrained after completed installation is in large part a function of the sub-surface soil conditions and behavior, and the soil-pressure at the depth of installation. Although the longitudinal displacement due to thermal expansion or contraction is minimal, the possibility of its displacement should be recognized. <u>The polyethylene pipe should be cut to length only after it is in thermal equilibrium with the surrounding soil (usually overnight).</u> In this way the “installed” versus “operating” Temperature difference is dropped to nearly zero and the pipe will have assumed its natural length at the existing soil/water temperature. Additionally, the thermal inertia of the pipe and soil will oppose any brief temperature changes from the flow stream. Seasonal temperature changes happen so slowly that actual thermally induced stresses are usually insignificant within polyethylene for design purposes.

TORSION STRESS

A typical value for torsional shear stress is 50% of the tensile strength. Divide the transmitted torque by the wall area to get the torsional shear stress intensity. During the pull-back and reaming procedure, a swivel is typically used to separate the rotating cutting head assembly from the pipeline pull segment. Swivels are not 100% efficient and some minor percent of torsion will be transmitted to the pipeline. For thick wall HDPE pipes of SDR 17, 15.5, 11, 9 and 7 this torsion is not significant and usually does not merit a detailed engineering analysis.

EXAMPLE CALCULATIONS

Example Calculations are given in the Appendix.

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APPENDIX A

DESIGN CALCULATION EXAMPLE FOR SERVIC LOADS (POST-INSTALLATION)

Example 1: A 6" IPS DR 11 HDPE pipe is being pulled under a railroad track. The minimum depth under the track is 10 ft. Determine the safety factor against buckling.

Given Parameters:

OD = 6.625 in	Nominal Pipe OD	DR = 11 Pipe	Dimension Ratio
H = 10 ft.	Max. Borehole Depth		
γ = 120 lbf/ft^3	Unit Weight of Soil	P_{Live} = 1100 lbf/ft^2	E-80 Live Load

PE Material Parameters:

Wheel loading from train will be applied for several minutes without relaxation. Repetitive trains crossing may accumulate. A conservative choice for the apparent modulus is the 1000-hour modulus.

E_{mid} = 43700 psi μ = 0.45 Long Term Poisson's Ration

Soil and Live Load Pressure on Pipe (Assuming that the earth load equals the prism load is perhaps too conservative except for a calculation involving dynamic surface loading.)

$P = (\gamma H + P_{Live})\ 1\ ft^2/144\ in^2$ P = 15.972 psi

Ring Deflection resulting from soil and live load pressures assuming no side support.

$$\%VD = \frac{0.0125\,P}{\left[\dfrac{E_{mid}}{12\,(DR-1)^3}\right]}$$

%VD = 5.482 Percent deflection from soil loads

Determine critical unconstrained buckling pressure based on deflection from loading and safety factor

f_o = 0.56 Ovality compensation factor for 5.5% ovality from Figure 3

$$P_{UC} = \frac{2E_{mid}}{(1 - m^2)} \left(\frac{1}{DR - 1} \right)^3 f_o$$

P_{UC} = 61.372 psi Critical unconstrained buckling pressure (no safety factor)

$$SF_{cr} = \frac{P_{UC}}{P}$$

SF_{cr} = 3 842 Safety factor against buckling

Example 2: A 6" IPS DR 13.5 HDPE pipe is being pulled under a small river for use as an electrical duct. At its lowest point, the pipe will be 18 feet below the river surface. Assume the slurry weight is equal to 75 Ib/cu.ft.. The duct is empty during the pull. Calculate a) the maximum pulling force and b) the safety factor against buckling for the pipe. Assume that the pipe's ovality is 3% and that the pulling time will not exceed 10 hours.

Solution:

Calculate the safe pull strength

OD = 6.625in. Pipe outside diameter

DR = 13.5 Pipe dimension ratio

T_{allow} = 1150 psi Typical safe pull stress for HDPE for 12-hour pull duration

$$F_s = pT_{allow}\, OD^2 \left(\frac{1}{DR} - \frac{1}{DR^2} \right)$$

$F_s = 1.088 \cdot 10^4$ lbf Safe pull strength for 6" IPS DR 13.5 HDPE pipe assuming 10-hour maximum pull duration

Step 1: Determine the critical buckling pressure during Installation for the pipe (include tensile reduction factor assuming the frictional drag during pull results in 1000 psi longitudinal pipe stress)

E = 57500 psi Apparent modulus of elasticity (for 10 hours at 73 degrees F)

m = 0.45 Poisson's ratio (long term value)

f_o = 0.76 Ovality compensation factor (for 3% ovality)

R = 0.435   Tensile ratio (based on assumed 1000 psi pull stress calculation)

$$f_R = \sqrt{5.57 - (r + 1.09)^2} - 1.09$$

$f_R = 0.711$ Tensile reduction factor

$$Pcr = \frac{2E}{(1 - \mu^2)} \left(\frac{1}{DR - 1} \right)^3 \cdot f_O \cdot f_R \qquad P_{Cr} = 39.907$$

Critical unconstrained buckling pressure for DR 13.5 pipe without safety factor

Step 2: Determine expected loads on pipe (assume only static drilling fluid head acting on pipe, and borehole intact with no soil loading)

$g_{slurry} = 75\ lbf/ft^3$ Drilling fluid weight   H = 18 ft   Maximum bore depth

$$P_{slurry} = Hg_{slurry} \left(\frac{1ft^2}{144in^2} \right) \qquad P_{slurry} = 9.375\ psi$$

Total static drilling fluid head pressure if drilled from surface

Step 3: Determine the resulting safety factor against critical buckling during installation

$$SF_{cr} = \frac{P_{cr}}{P_{slurry}} \qquad SF_{cr} = 4.257$$

Safety factor against critical buckling during pull. Most designers consider a SF of 2 adequate for buckling.

Example 3: Determine the safety factor for long term performance for the communication duct in example 2. Assume there are 10 feet of riverbed deposits above the borehole having a saturated unit weight of 110 lb/cu.ft.. (18 feet deep, 3% initial ovality)

Solution:

Step 1: Determine the pipe soil load (Warning requires Input of ovality compensation in step 4.)

E long = 28200 psi	Long term apparent modulus
$g_w = 62.4\ lbf/ft.^3$	Unit weight of water
H = 18 ft	Max. borehole depth
$g_s = 110\ lbf/ft.^3$	Saturated unit weight of sediments

GW = 18 ft	Groundwater height
C = 10ft.	Height of soil cover
OD = 6.625 in	Nominal pipe OD
DR = 13.5	Pipe dimension ratio
m = 0.45	Long Term Poisson's ratio

$$P_{soil} = (\gamma_{ss} - \gamma_w)\, C \left(\frac{1\ ft^2}{144\ in^2}\right) \qquad P_{soil} = 3.306 \text{ psi}$$

Prism load on pipe from10' of saturated cover (including buoyant force on submerged soil)

Step 2: Calculate the ring deflection. Use the larger of the deflection resulting from (a) soil loads assuming no side support or from (b) buoyant deflection due to mud weight.

$$\%\Delta V_b = \frac{0.0125 \cdot P_{soil}}{\left[\frac{E_{long}}{12\,(DR-1)^3}\right]} \cdot 100 \qquad \%\Delta V_b = 3.434$$

Percent deflection from soil loads

t = OD/DR $\qquad$ t =0.491 in

$$\%\Delta V_b = \frac{0.2337\gamma_{SS}\left(\frac{OD - 1.06t}{2}\right)^4}{E_{long}\left(\frac{t^3}{12}\right)} 100 \qquad \%\Delta V_b = 0.465$$

Percent deflection form buoyancy force

Step 3: Determine the long-term hydrostatic loads on the pipe

$$P_W = \left(\frac{GW}{2.31 ft/psi}\right) + P_{soil} \qquad P_W = 11.098$$

External pressure due to groundwater head

g_{slurry} = 75 lb/cu.ft.^{3} $\qquad$ Unit weight of drilling fluid

$$P_{slurry} = g_{slurry}\, H \left(\frac{1ft^2}{144in^2}\right) \qquad P_{slurry} = 9.375 \text{ psi}$$

External pressure due to slurry head

$P_W > P_{slurry}$, therefor use P_W for buckling load

Step 4: Determine critical unconstrained buckling pressure based on deflection from loading

$f_o = 0.64$ 5% Ovality Compensation based on 3% initial ovality and 2% deflection

$$P_{UC} = \frac{2E_{long}}{(1-\mu^2)}\left(\frac{1}{DR-1}\right)^3 f_o \qquad P_{UC} = 23.174\text{psi}$$

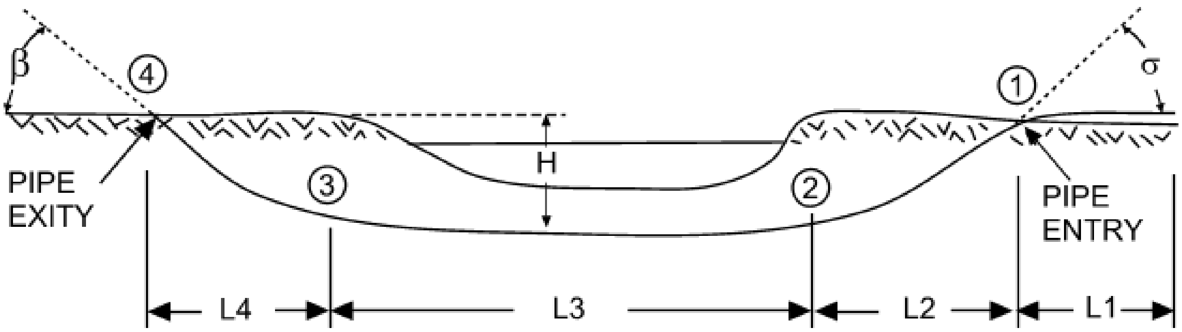
Critical unconstrained buckling pressure (no safety factor)

$$SF_{CR} = \frac{P_{UC}}{P_W} \qquad SF_{CR} = 2.088$$

Safety factor against buckling pressure of highest load (slurry)

APPENDIX B:

DESIGN CALCULATIONS EXAMPLE FOR PULL-BACK FORCE



Example 1: Find the estimated force required to pull-back pipe for the above theoretical river crossing using Slavin's Method. Determine the safety factor against collapse. Assume the HDPE pipe is 35 ft deep and approximately 870 ft long with a 10 deg. entry angle and a 15 deg. exit angle. Actual pull-back force will vary depending on backreamer size, selection, and use; bore hole staying open; soil conditions; lubrication with betonnite; driller expertise; and other application circumstances.

Pipe Properties

Outside Diameter:	OD = 24 in	Long-term Modulus	E_{long} = 28250 psi
Standard Dimension Ratio	DR = 11	24 hr Modulus	E_{24hr} = 56500 psi
Minimum wall thickness	t = 2.182 in	Poisson's ratio (long term)	m = 0.45
	Safe Pull Stress (24 hr)		s_{pb} = 1100 psi

Path Profile:

H = 35 ft	Depth of bore
q_{in} = 10 deg	Pipe entry angle
q_{ex} = 15 deg	Pipe exit angle
L_1 = 100 ft	Pipe drag on surface (This value starts at total length of pull, approximately 870 ft. then decreases with time. Assume 100 fit remaining at end of pull)
L_{cross} = 870 ft	

Path length (Determine L2 and L4):

Average Radius of Curvature for Path at Pipe Entry   q is given in radians

$R_{avgin} = 2H/q_{in}^2$ $R_{avgin} = 2.298 \cdot 10^3 ft$

Average Radius of Curvature for Path at Pipe Exit

$R_{agex} = 2H/q_{ex}^2$ $R_{agex} = 1.021 \cdot 10^3 ft$

Horizontal Distance Required to Achieve Depth or Rise to the Surface at Pipe Entry

$L_2 = 2H/q_{in}$ $L_2 = 401.07$ ft

Horizontal Distance Required to Achieve Depth or Rise to the Surface at Pipe Exit

$L_4 = 2H/q_{ex}$ $L_4 = 267.38$ ft

Where: L_2 & L_4 = horizontal transition distance at bore exit & entry respectively.

Determine Axial Bending Stress:

$R = R_{avgex}$ Min. Radius for Drill path

$R = 1.021 \cdot 10^3$ fl

OD = 24 in

Radius of curvature should exceed 40 times the pipe outside diameter to prevent ring collapse.

r = 40 OD

r = 80 ft   Okay. R > r

Bending strain

$e_a = OD/2R$ $e_a = 9.791 \cdot 10^{-4}$ in/in

Where: e_a = bending strain, in/in

OD = outside diameter of pipe, in

R = minimum radius of curvature, ft

Bending stress

$s_a = E_{24hr} e_a$ $s_a = 55.321$ psi

where s_a = bending stress, psi

Find Pulling Force:

Weight of Empty Pipe

$P_w = 3.61 \cdot 10^{-2}$ lbf/in³

$g_a = 0.95$ $\qquad g_b = 1.5$

$w_a = pOD^2\ (DR\text{-}1/DR^2) r_w\ g_a\ 12$ in/ft $\qquad w_a = 61.546$ lbf/ft

Net Upward Buoyant Force on Empty Pipe Surrounded by Mud Slurry

$W_b = p(OD^2/4)\ r_w\ g_{b\,-}\ w_a$ $\qquad w_b = 232.417$ lbf/ft

where: r_w = density of water, lb/in^3

g_a = specific gravity of the pipe material

g_b = specific gravity of the mud slurry

w_a = weight of empty pipe, lbf/ft

w_b = net upward buoyant force on empty pipe surrounded by mud slurry

Determine pull-back force acting on pipe

See figure:

$L_1 = 100$ ft	$v_a = 0.4$	
$L_2 = 401.07$ft	$v_b = 0.25$	
$L_3 = 200$ft	$a = q_{in}$	$a = 10$ deg
$L_4 = 267.38$	$b = q_{in}$	$b = 15$ deg
$L_3 = L_{cross} - L_2 - L_4$	$L_3 = 201.549$.ft	

$T_A = \exp(v_a\ a)\ [v_a\ w_a\ (L_1 + L_2 + L_3 + L_4)]$

$T_A = 2.561 \cdot 10^4$ lbf

$T_B = \exp(v_b\ a)\ (T_A + v_b\ [w_b]\ L_2 + w_b\ H - v_a\ w_a\ L_2 \exp(v_b\ a))$

$T_B = 4.853 \cdot 10^4$ lbf

$T_C = T_B + v_b\ [w_b]\ L_3 - \exp(v_b\ a)\ (v_a\ w_a\ L_3.\exp(v_a\ a))$

$T_C = 5.468 \cdot 10^4$ lbf

$T_D = \exp(v_b\ b)\ [T_C + v_b\ [w_b]\ L_4 - w_b\ H - \exp(v_b\ a)\ (v_a\ w_a\ L_4 \exp(v_b\ a))]$

$T_D = 5.841 \cdot 10^4$ lbf

Where: T_A = pull force on pipe at point A, lbf

T_B = pull force on pipe at point B. lbf

T_C = pull force on pipe at point C, lbf

T_D = pull force on pipe at point D, lbf

L_1 = pipe on surface, ft

L_2 = horizontal distance to achieve desired depth, ft

L_3 = additional distance traversed at desired depth, ft

L_4 = horizontal distance to rise to surface, ft

v_a = coefficient of friction applicable at the surface before the pipe enters bore hole

v_b = coefficient of friction applicable within the lubricated bore hole or after the (wet) pipe exits

a = bore hole angle at pipe entry, radians

b = bore hole angle at pipe exit, radians

(refer to figure 1)

Hydrokineti Pressure

$DP = 10$ psi

$D_h = 1.5$ 0D $\qquad D_h = 36.$in

$DT = DP\ (p/8)\ (D_h^2 - OD^2) \qquad DT = 2.827 \cdot 10^3 \cdot$lbf

Where: DT = pulling force increment, lbf

DP = hydrokentetic pressure, psi

D_h = back reamed hole diameter, in

Compare Axial Tensile Stress with Allowable Tensile Stress During Pull-back of 1100 psi:

Compare Axial Tensile Stress with Allowable Tensile Stress During Pull-back of 1100 psi

Average Axial Stress Acting on Pipe Cross-section at Points A, B. C, D

$$s_i = (Ti + DT) \left(\frac{1}{pOD^2}\right) \left(\frac{DR^2}{DR-1}\right)$$

s_1 = 190.13 psi <1100 psi OK

s_2 = 343.408 psi <1100 psi OK

s_3 = 384.551 psi <1100 psi OK

s_4 = 409.484 psi <1100 psi OK

Where: $Ti = T_A, T_B, T_C, T_D$ (lbf)

s_i = corresponding stress, psi

Breakaway links should be set so that pull-back force applied to pipe does not exceed 1100 psi stress.

ID = OD - 2t

$F_b = s_{pb}\ (p/4)(OD^2 - ID^2)$ $\quad F_b = 1.645 \cdot 10^5 \cdot lbf$

Determine safety factor against ring collapse during pull-back

External Hydraulic Load

External static head pressure

$P_{ha} = (1.5)\ (62.4\ lbf/ft^3)\ (H)$ $\quad P_{ha} = 22.75.psi$

Combine static head with hydrokinetic pressure

$P_{effa} = P_{ha} + DP$ $\quad P_{effa} = 32.75$ psi

Critical collapse pressure

Resistance to external hydraulic load during pull-back

$f_o = 0.76$ Ovality compensation factor (for 3% ovality)

$r = s_4/2s_{pb}$ $\quad r = 0.186$ Tensile ratio (based on assumed 1100 psi pull stress calculation)

$f_R = \sqrt{5.57 - (r + 1.09)^2} - 1.09$ $\quad f_R = 0895$ Tensile reduction factor

$$P_{CR} = \frac{2E_{24hr}}{(1 - \mu^2)} \left(\frac{1}{DR - 1} \right)^3 f_o \, f_R \qquad P_{CR} = 96.414 \text{ psi}$$

Safety factor against collapse

$SF = P_{cr}/P_{ha}$ $\qquad$ $SF = 4.238$

Where: P_{ha} = applied efflective pressure due to head of water of drilling

P_{cr} = calculated critical buckling pressure due to head of water of drilling fluid, psi

SF = Safety Factor

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